

## DAMAGE DETECTION II

### *Field Applications to Large Structures*

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### 1. Introduction

In the most general terms damage can be defined as changes introduced into a system that adversely effect the current or future performance of that system. Implicit in this definition is the concept that damage is not meaningful without a comparison between two different states of the system, one of which is assumed to represent the initial, and often undamaged, state. This discussion is focused on the study of damage identification in structural and mechanical systems. Therefore, the definition of damage will be limited to changes to the material and/or geometric properties of these systems, including changes to the boundary conditions and system connectivity, which adversely effect the current or future performance of that system.

The interest in the ability to monitor a structure and detect damage at the earliest possible stage is pervasive throughout the civil, mechanical and aerospace engineering communities. Current damage-detection methods are either visual or localized experimental methods such as acoustic or ultrasonic methods, magnetic field methods, radiograph, eddy-current methods and thermal field methods (Doherty, 1987). All of these experimental techniques require that the vicinity of the damage is known *a priori* and that the portion of the structure being inspected is readily accessible. Subjected to these limitations, these experimental methods can detect damage on or near the surface of the structure. The need for quantitative global damage detection methods that can be applied to complex structures has led to the development and continued research of methods that examine changes in the vibration characteristics of the structure.

The basic premise of vibration-based damage detection is that the damage will significantly alter the stiffness, mass or energy dissipation properties of a system, which, in turn, will alter the measured dynamic response of that system. Although the basis for vibration-based damage detection appears intuitive, its actual application poses many significant technical challenges. The most fundamental challenge is the fact that damage is typically a local phenomenon and may not significantly influence the lower-frequency global response of structures that is typically measured during vibration tests. This challenge is supplemented by many practical issues associated with making accurate and repeatable vibration measurements at a limited number of locations on structures often operating in adverse environments.

In an effort to emphasize the extent of the research efforts in vibration-based damage detection a brief summary of applications that have driven developments in this field over the last thirty years is first presented. Recent research has begun to recognize that the vibration-based damage detection problem is fundamentally one of statistical pattern recognition and this paradigm is described in detail. Current damage detection methods are then summarized in the context of this paradigm.

Finally, this paper will discuss the application of this technology to large civil engineering structures. The destructive tests performed on the I-40 Bridge over the Rio Grande in Albuquerque, NM are used as the example. This structure was chosen because numerous investigators have independently applied a variety of damage detection methods to the data obtained from these tests. To the authors' knowledge, this represents one of the most studied data sets from a test specifically aimed at detecting damage in an *in situ* structure from changes in its vibration characteristics. A study of the various damage detection methods that have been applied to this structure provides a better understanding of the methods

themselves and many issues associated with the actual implementation of this technology. The paper concludes by describing a damage detection investigation of concrete columns in terms of the statistical pattern recognition paradigm.

## **2. Historical Developments**

It is the authors' speculation that damage or fault detection, as determined by changes in the dynamic properties or response of systems, has been practiced in a qualitative manner, using acoustic techniques, since modern man has used tools. More recently, this subject has received considerable attention in the technical literature and a brief summary of the developments in this technology over the last thirty years is presented below. Specific references are not cited; instead the reader is referred to (Doebling, et al. 1996) for a review of literature on this subject.

The development of vibration-based damage detection technology has been closely coupled with the evolution, miniaturization and cost reductions of Fast Fourier Transform (FFT) analyzer hardware and computing hardware. To date, the most successful application of vibration-based damage detection technology has been for monitoring rotating machinery. The rotating machinery application has taken an almost exclusive non-model based approach to damage detection. The detection process is based on pattern recognition applied to time histories or spectra generally measured on the housing of the machinery during normal operating conditions. Databases have been developed that allow specific types of damage to be identified from particular features of the vibration signature. For these systems the approximate location of the damage is generally known making a single channel FFT analyzer sufficient for most periodic monitoring activities. Today, commercial software integrated with measurement hardware is marketed to help the user systematically apply this technology to operating equipment.

During the 1970s and 1980s the oil industry made considerable efforts to develop vibration-based damage detection methods for offshore platforms. This damage detection problem is fundamentally different from that of rotating machinery because the damage location is unknown and because the majority of the structure is not readily accessible for measurement. To circumvent these difficulties, a common methodology adopted by this industry was to simulate candidate damage scenarios with numerical models, examine the changes in resonant frequencies that were produced by these simulated changes, and correlate these changes with those measured on a platform. A number of very practical problems were encountered including measurement difficulties caused by platform machine noise, instrumentation difficulties in hostile environments, changing mass caused by marine growth and varying fluid storage levels, temporal variability of foundation conditions, and the inability of wave motion to excite higher modes. These issues prevented adaptation of this technology and efforts at further developing this technology for offshore platforms were largely abandoned in the early 1980s.

The aerospace community began to study the use of vibration-based damage detection during the late 1970's and early 1980's in conjunction with the development of the space shuttle. This work has continued with current applications being investigated for the National Aeronautics and Space Administration's space station and reusable launch vehicle. The Shuttle Modal Inspection System (SMIS) was developed to identify fatigue damage in components such as control surfaces, fuselage panels and lifting surfaces. These areas were covered with a thermal protection system making these portions of the shuttle inaccessible and, hence impractical for conventional local non-destructive examination methods. This system has been successful in locating damaged components that are covered by the thermal protection system. All orbiter vehicles have been periodically subjected to SMIS testing since 1987. Space station applications have primarily driven the development of experimental/analytical damage detection methods. These approaches are based on correlating analytical models of the undamaged structure with measured modal properties from both the undamaged and damaged structure. Changes in stiffness indices as assessed from the two model updates are used to locate and quantify the damage. Since the mid 1990's, studies of damage detection for composite materials have been motivated by the development composite fuel tank for a reusable launch vehicle.

The civil engineering community has studied vibration based damage assessment of bridge structures since the early 1980's. Modal properties and quantities derived from these properties such as mode-shape curvature and dynamic flexibility matrix indices have been the primary features used to identify damage in bridge structures. Environmental and operating condition variability present significant challenges to the bridge monitoring application. Regulatory requirements in eastern Asian countries, which mandate the companies that construct the bridges to periodically certify their structural health, are driving current research and development of vibration-based bridge monitoring systems.

In summary, the review of the technical literature presented by (Doebeling et al. 1996) shows an increasing number of research studies related to vibration-based damage detection. These studies identify many technical challenges to the adaptation of vibration-based damage detection that are common to all applications of this technology. These challenges include better utilizing the nonlinear response characteristics of the damaged system, development of methods to optimally define the number and location of the sensors, identifying the features sensitive to small damage levels, the ability to discriminate changes in features cause by damage from those caused by changing environmental and/or test conditions, the development of statistical methods to discriminate features from undamaged and damaged structures, and performing comparative studies of different damage detection methods applied to common data sets. These topics are currently the focus of various research efforts by many industries including defense, automotive, and semiconductor manufacturing where multi-disciplinary approaches are being used to advance the current capabilities of vibration-based damage detection.

### **3. Vibration-Based Damage Detection and Structural Health Monitoring**

The process of implementing a damage detection strategy is referred to as *structural health monitoring*. This process involves the observation of a structure over a period of time using periodically spaced measurements, the extraction of features from these measurements, and the analysis of these features to determine the current state of health of the system. The output of this process is periodically updated information regarding the ability of the structure to continue to perform its desired function in light of the inevitable aging and degradation resulting from the operational environments. Figure 1 shows a chart summarizing the structural health-monitoring process. The topics summarized in this figure are discussed below.

#### **3. 1. OPERATIONAL EVALUATION**

Operational evaluation answers two questions in the implementation of a structural health monitoring system:

1. What are the conditions, both operational and environmental, under which the system to be monitored functions?
2. What are the limitations on acquiring data in the operational environment?

Operational evaluation begins to set the limitations on what will be monitored and how the monitoring will be accomplished. This evaluation starts to tailor the damage detection process to features that are unique to the system being monitored and tries to take advantage of unique features of the postulated damage that is to be detected.

#### **3. 2. DATA ACQUISITION AND CLEANSING**

The data acquisition portion of the structural health monitoring process involves selecting the types of sensors to be used, the location where the sensors should be placed, the number of sensors to be used, and the data acquisition/storage/transmittal hardware. This process will be application specific. Economic considerations will play a major role in making these decisions. Another consideration is how often the data should be collected. In some cases it may be adequate to collect data immediately before and at

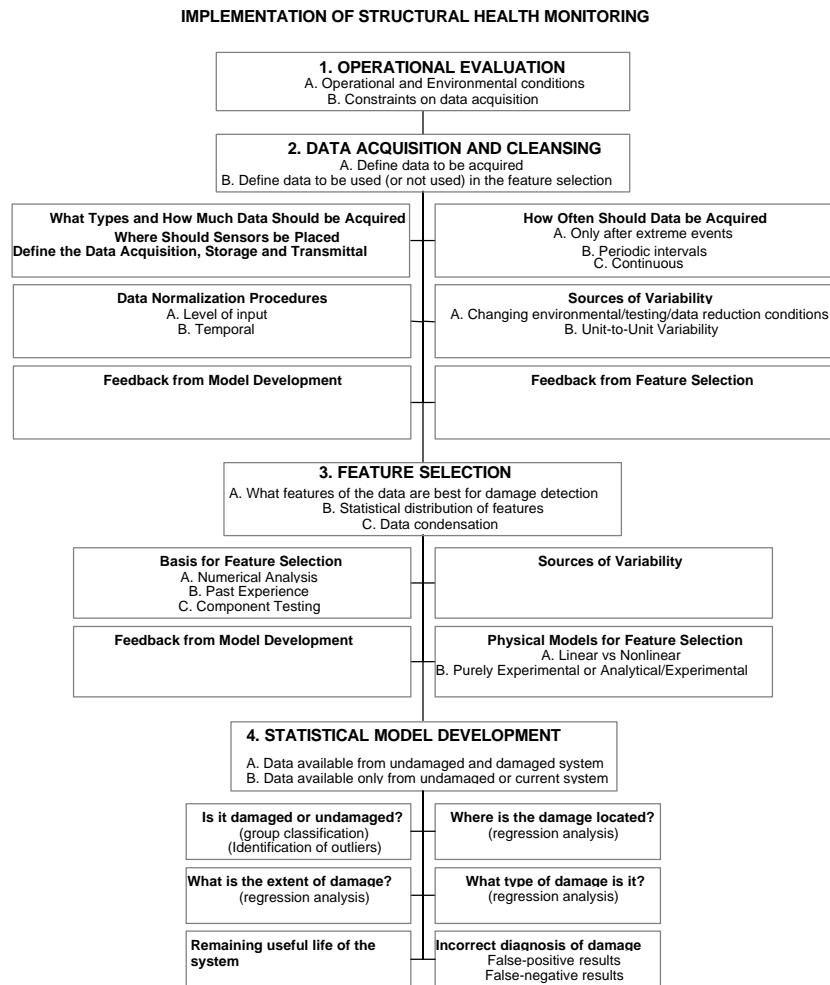


Fig. 1 Flow chart for implementing a structural health monitoring program.

periodic intervals after a severe event. However, if fatigue crack growth is the failure mode of concern, it may be necessary to collect data almost continuously at relatively short time intervals.

Because data can be measured under varying conditions, the ability to normalize the data becomes very important to the damage detection process. One of the most common procedures is to normalize the measured responses by the measured inputs. When environmental or operating condition variability is an issue, the need can arise to normalize the data in some temporal fashion to facilitate the comparison of data measured at similar times of an environmental or operational cycle. Sources of variability in the data acquisition process and with the system being monitored need to be identified and minimized to the extent possible. In general, all sources of variability can not be eliminated. Therefore, it is necessary to make the appropriate measurements such that these sources can be statistically quantified.

Data cleansing is the process of selectively choosing data to accept for, or reject from, the feature selection process. The data cleansing process is usually based on knowledge gained by individuals directly involved with the data acquisition. Finally, it should be noted that the data acquisition and cleansing portion of a structural health-monitoring process should not be static. Insight gained from the feature selection process and the statistical model development process will provide information regarding changes that can improve the data acquisition process.

### 3. 3. FEATURE SELECTION

The area of the structural damage detection process that receives the most attention in the technical literature is the identification of data features that allow one to distinguish between the undamaged and damaged structure. Inherent in this feature selection process is the condensation of the data. The operational implementation and diagnostic measurement technologies needed to perform structural health monitoring typically produce a large amount of data. A condensation of the data is advantageous and necessary particularly if comparisons of many data sets over the lifetime of the structure are envisioned. Also, because data may be acquired from a structure over an extended period of time and in an operational environment, robust data reduction techniques must retain sensitivity of the chosen features to the structural changes of interest in the presence of environmental noise.

The best features for damage detection are typically application specific. Numerous features are often identified for a structure and assembled into a feature vector. In general, it is desirable to develop feature vectors that are of low dimension. It is also desirable to obtain many samples of the feature vectors. There are no restrictions on the types or combinations of data contained in the feature vector. As an example, a feature vector may contain the first three resonant frequencies of the system, a time when the measurements were made, and a temperature reading from the system. A variety of methods are employed to identify features for damage detection. Past experience with measured data from a system, particularly if damaging events have been previously observed for that system, is often the basis for feature selection. Numerical simulation of the damaged system's response to simulated inputs is another means of identifying features for damage detection. The application of engineered flaws, similar to ones expected in actual operating conditions, to specimens can identify parameters that are sensitive to the expected damage. Damage accumulation testing, during which significant structural components of the system under study are subjected to a realistic accumulation of damage, can also be used to identify appropriate features. Fitting linear or nonlinear, physical-based or non-physical-based models of the structural response to measured data can also help identify damage-sensitive features. Common features used in vibration-based damage detection studies are briefly summarized below. A more detailed summary can be found in (Doebeling, et al., 1996).

#### 3.3.1 Basic Modal Properties

The most common features that are used in vibration-based damage detection, and that represent a significant amount of data condensation from the actual measured quantities, are the common modal properties of resonant frequencies and mode-shape vectors. These features are identified from measured response time-histories, most often absolute acceleration, or spectra of these time-histories. The technology required to accurately make these measurements is summarized in (McConnell, 1995). Often these spectra are normalized by spectra of the measured force input to form frequency response functions. Well-developed experimental modal analysis procedures are applied to these functions or to the measured-response spectra to estimate the system's modal properties (Ewins, 1995, and Maia and Silva, 1997).

The amount of literature that uses resonant frequency shifts as a data feature for damage detection is quite large. The observation that changes in structural properties cause changes in vibration frequencies was a primary impetus for developing vibration-based damage identification technology. In general, changes in frequencies cannot provide spatial information about structural changes. For applications to large civil engineering structures the somewhat low sensitivity of frequency shifts to damage requires either very precise measurements of frequency change or large levels of damage. An exception to this limitation occurs at higher modal frequencies, where the modes are associated with local responses. However, the practical limitations involved with the excitation and identification of the resonant frequencies associated with these local modes, caused in part by high modal density and low participation factors, can make them difficult to identify.

Damage detection methods using mode shape vectors as a feature generally analyze differences between the measured modal vectors before and after damage. Mode shape vectors are spatially

distributed quantities; therefore, they provide information that can be used to locate damage. However, large number of measurement locations can be required to accurately characterize mode shape vectors and provide sufficient resolution for determining the damage location.

### 3.3.2. *Mode Shape Curvature Changes*

An alternative to using mode shapes to obtain spatially distributed features sensitive to damage is to use mode shape derivatives, such as curvature. Mode shape curvature can be computed by numerically differentiating the identified mode shape vectors twice to obtain an estimate of the curvature. These methods are motivated by the fact that the second derivative of the mode shape is much more sensitive to small perturbations in the system than is the mode shape itself. Also, for beam- and plate-like structures changes in curvature can be related to changes in strain energy, which has been shown to be a sensitive indicator of damage. A comparison of the relative statistical uncertainty associated with estimates of mode shape curvature, mode shape vectors and resonant frequencies showed that the largest variability is associated with estimates of mode shape curvature followed by estimates of the mode shape vector. Resonant frequencies could be estimated with least uncertainty (Doebeling, Farrar and Goodman, 1997).

### 3.3.3. *Dynamically Measured Flexibility*

Changes in the dynamically measured flexibility matrix indices have also been used as damage sensitive features. The dynamically measured flexibility matrix is estimated from the mass-normalized measured mode shapes and measured eigenvalue matrix (diagonal matrix of squared modal frequencies). The formulation of the flexibility matrix is approximate because in most cases all of the structure's modes are not measured. Typically, damage is detected using flexibility matrices by comparing the flexibility matrix indices computed using the modes of the damaged structure to the flexibility matrix indices computed using the modes of the undamaged structure. Because of the inverse relationship to the square of the modal frequencies, the measured flexibility matrix is most sensitive to changes in the lower-frequency modes of the structure.

### 3.3.4. *Updating Structural Model Parameters*

Another class of damage identification methods is based on features related to changes in mass, stiffness and damping matrix indices that have been correlated such that the numerical model predicts as closely as possible to the identified dynamic properties (resonant frequencies, modal damping and mode shape vectors) of the undamaged and damaged structures, respectively. These methods solve for the updated matrices (or perturbations to the nominal model that produce the updated matrices) by forming a constrained optimization problem based on the structural equations of motion, the nominal model, and the identified modal properties (Friswell and Mottershead, 1995). Comparisons of the matrix indices that have been correlated with modal properties identified from the damaged structure to the original correlated matrix indices provide an indication of damage that can be used to quantify the location and extent of damage. Degree of freedom mismatch between the numerical model and the measurement locations can be a severe limitation for performing the required matrix updates.

### 3.3.5. *Nonlinear Methods*

Identification of the previously described features is based on the assumption that a linear modal can be used to represent the structural response before and after damage. However, in many cases the damage will cause the structure to exhibit nonlinear response. Therefore, the identification of features indicative of nonlinear response can be a very effective means of identifying damage in a structure that originally exhibited linear response. The specific features that indicate a system is responding in a nonlinear manner vary widely. Examples include the generation of resonant frequency harmonics in a cracked beam excited in a manner such that the crack opens and closes (Prime and Shevitz, 1996). For extreme event such as earthquake, the normalized arias intensity provides an estimate of kinetic energy of the structure and has been successfully used to identify the onset of nonlinear response buildings subject to damaging earthquake excitations (Straser, 1998). Deviations from a Gaussian probability distribution function of acceleration response amplitudes for a system subjected to a Gaussian input have been used

successfully to identify that loose parts are present in a system. Temporal variation in resonant frequencies as identified with canonical variate analysis is another method to identify the onset of damage (Hunter, 1999). In general, features based on the nonlinear response of a system have only been used to identify that damage has occurred. Few methods have been described that locate the source of the nonlinearity. Because all systems will exhibit some degree of nonlinearity, a challenge becomes to establish a threshold at which changes in the nonlinear response features are indicative of damage. The statistical model building portion of the structural health monitoring process is essential for establishing such thresholds.

### 3. 4. STATISTICAL MODEL DEVELOPMENT

The portion of the structural health monitoring process that has received the least attention in the technical literature is the development of statistical models to enhance the damage detection. Almost none of the hundreds of studies summarized in (Doebeling, et al, 1996) make use of any statistical methods to assess if the changes in the selected features used to identify damaged are statistically significant. Statistical model development is concerned with the implementation of the algorithms that operate on the extracted features and unambiguously determine the damage state of the structure. The algorithms used in statistical model development usually fall into three categories and will depend on the availability of data from both an undamaged and damaged structure. The first category is group classification, that is, placement of the features into respective “undamaged” or “damaged” categories. Analysis of outliers is the second type of algorithm. When data from a damaged structure are not available for comparison, do the observed features indicate a significant change from the previously observed features that can not be explained by extrapolation of the feature distribution? The third category is regression analysis. This analysis refers to the process of correlating data features with particular types, locations or extents of damage. All three algorithm categories analyze statistical distributions of the measured or derived features to enhance the damage detection process.

The statistical models are used to answer the following questions regarding the damage state of the structure, (Rytter, 1993): 1. Is there damage in the structure (existence)?; 2. Where is the damage in the structure (location)?; and 3. How severe is the damage (extent)? Answers to these questions in the order presented represents increasing knowledge of the damage state. Experimental structural dynamics techniques can be used to address the first two questions. Analytical models are usually needed to answer the third question unless examples of data are available from the system (or a similar system) when it exhibits varying level of the damage. Statistical model development can also determine the type of damage that is present. To identify the type of damage, data from structures with the specific types of damage must be available for correlation with the measured features.

Finally, an important part of the statistical model development process is the testing of these models on actual data to establish the sensitivity of the selected features to damage and to study the possibility of false indications of damage. False indications of damage fall into two categories: 1.) False-positive damage indication (indication of damage when none is present), and 2). False-negative damage indications (no indication of damage when damage is present). Although the second category is usually very detrimental to the damage detection and can have serious life-safety implications, false-positive readings can also erode confidence in the damage detection process.

This paper will now summarize a damage detection study performed on an *in situ* bridge. This summary is followed by the application of methods from statistical pattern recognition and machine learning to the vibration-based damage detection study of concrete columns. A damage detection experiment performed on concrete bridge columns will be described in terms of the statistical-pattern-recognition damage-detection paradigm that has just been summarized.

## 4. I-40 Bridge Damage Detection Study

The Interstate 40 (I-40) Bridges over the Rio Grande in Albuquerque, New Mexico were scheduled to be

demolished in the summer of 1993 as part of a construction project to widen the bridges for increased traffic flow. Prior to this demolition, damage was incrementally applied to the bridge in a controlled manner. Vibration tests were performed on the undamaged bridge and after each level of damage had been introduced. The primary purpose of these tests was to study the ability to identify and locate damage in a large structure based on changes in its measured dynamic response.

#### 4.1. TEST STRUCTURE GEOMETRY

The I-40 Bridges formerly consisted of twin spans (there are separate bridges for each traffic direction) made up of a concrete deck supported by two welded-steel plate girders and three steel stringers. Loads from the stringers were transferred to the plate girders by floor beams located at 6.1-m (20-ft) intervals. Cross-bracing was provided between the floor beams. Fig. 1 shows an elevation view of the portion of the bridge that was tested. The cross-section geometry of each bridge is shown in Fig. 2. Each bridge was made up of three identical sections. Each section had three spans; the end spans were of equal length, approximately 39.9 m (131 ft), and the center span was approximately 49.4-m (163 ft) long. All subsequent discussions of the I-40 Bridge will refer to the bridge carrying eastbound traffic, particularly the three eastern spans, which were the only ones tested.

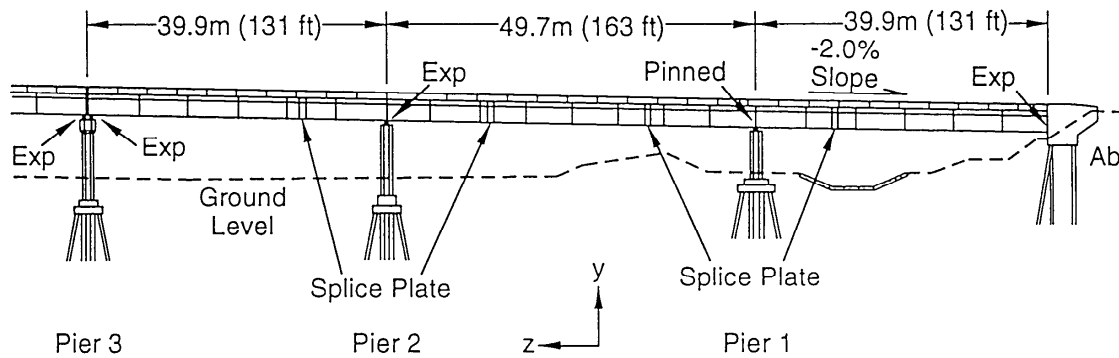


Fig. 1. Elevation view of the portion of the I-40 Bridge that was tested.

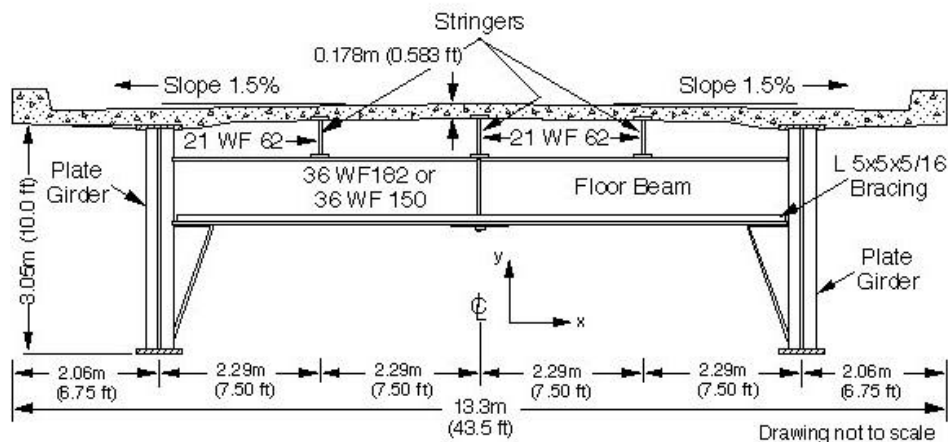


Fig. 2. Cross-section geometry of the I-40 Bridge.

## 4.2. DAMAGE SCENARIOS

The damage that was introduced was intended to simulate fatigue cracking that has been observed in plate-girder bridges. This type of cracking results from out-of-plane bending of the plate girder web and usually begins at welded attachments to the web such as the seats supporting the floor beams. Four levels of damage were introduced to the middle span of the north plate girder close to the seat supporting the floor beam at mid-span. Damage was introduced by making various torch cuts in the web and flange of the girder. A major drawback of introducing damage in this manner is that the torch cuts produce crack much wider than an actual fatigue crack. Hence, these cracks do not open and close under the loading that was applied during this study.

The first level of damage, designated E-1, consisted of a 0.61-m-long (2-ft-long), 10-mm-wide (3/8-in-wide) cut through the web centered at mid-height of the web. Next, this cut was continued to the

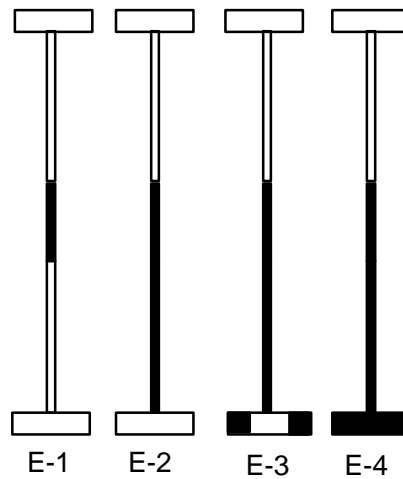


Fig. 3 Damage Scenarios

bottom of the web to produce a second level of damage designated E-2. For the third level of damage, E-3, the flange was then cut halfway in from either side directly below the cut in the web. Finally, the flange was cut completely through for damage case E-4 leaving the top 1.2 m (4 ft) of the web and the top flange to carry the load at this location. The various levels of damage are shown in Fig. 3. Photographs of the damage can be found in (Farrar, et al., 1994).

## 4.3 OPERATIONAL EVALUATION

Safety concerns did not allow traffic on the bridge after damage had been introduced. Therefore, all testing in the damaged condition required a shaker to be used as the excitation source. Safety and time constraints dictated that the shaker could only be setup in one location and only on the span closest to the abutment. The data acquisition system used limited the number of channels that could be acquired simultaneously to 32. Access to the girders under the bridge by catwalks limited the locations that accelerometers could be placed to the two main plate girders. Considerable extraneous vibration inputs were caused by traffic on the adjacent spans. Because the contractor was anxious to proceed with the demolition they began to tear down the bridge being tested at its other end and to remove fill near the abutment that was being tested. These activities add additional sources of variability into the testing that are difficult to quantify. The testing was constrained by budget and scheduling concerns to be completed during daylight hours over a two-week time window during the hottest time of the year. Electric power was not available during these tests.

#### 4.4. DATA ACQUISITION AND CLEANSING

The experimental procedures used to obtain vibration response data from the undamaged and damaged structure are described in this section. To obtain these data, a forced vibration test was conducted on the undamaged bridge. Forced vibration tests similar to those done on the undamaged structure were repeated after each level of damage had been introduced. A detailed summary of the experimental procedures can be found in (Farrar et al., 1994).

Engineers from Sandia National Laboratory (SNL) provided a hydraulic shaker that generated the measured force input (Mayes and Nusser, 1994). A random-signal generator was used to produce a 8.90 kN (2000-lb) peak-force uniform random signal over the frequency range of 2 to 12 Hz. An accelerometer mounted on the reaction mass was used to measure the force input to the bridge. The shaker was located on the eastern-most span directly above the south plate girder and midway between the abutment and first pier.

The data acquisition system used in these tests consisted of a computer workstation that controlled 29 input modules and a signal-processing module. The workstation was also the platform for a commercial data-acquisition/signal-analysis/modal-analysis software package. The input modules provided power to the accelerometers and performed analog-to-digital conversion of the accelerometer voltage-time histories. The signal-processing module performed the needed fast Fourier transform calculations.

Two sets of integral-circuit, piezoelectric accelerometers were used for the vibration measurements. A coarse set of measurements (SET1) was first made. These accelerometers were mounted in the vertical direction, on the inside web of the plate girder, at mid-height and at the axial locations shown in Fig. 4. A more refined set of measurements (SET2) was made near the damage location. Eleven accelerometers were placed in the global Y direction at a nominal spacing of 4.88 m (16 ft) along the mid-span of the north plated girder. All accelerometers were located at mid-height of the girder. The spacing of these accelerometers relative to the damage is shown in Fig. 5.

The only form of data cleansing that was performed was to use the overload reject feature in the data acquisition system. All accelerometers and their corresponding wiring were checked before and after each test to verify that they were working properly. In instances when a faulty accelerometer or wire was detected, the problem was corrected and the test was repeated.

#### 4.5. FEATURE EXTRACTION AND DATA COMPRESSION

Basic modal parameters (resonant frequencies, mode shapes and modal damping values) were used as features as well as properties derived from these quantities. Also, stiffness indices from updated finite element models were used as damage sensitive features. The reduction of the measured accelerometer time histories to modal parameters represents a significant amount of data compression. A typical time-history for the I-40 Bridge test had 2048 data points, and if measurements are made at 26 points plus an input, there are 55,296 pieces of information regarding the current state of the structure. Through system identification procedures commonly referred to as experimental modal analysis this volume of data was reduced to six resonant frequencies, mode shapes and modal damping values. This data compression was done because the modal quantities are easier to visualize, physically interpret, and interpret in terms of standard mathematical modeling of vibrating systems than are the actual time-history measurements. Therefore, the 55,296 pieces of information were reduced to 324 pieces of information (6 modes made up of 26 amplitude and phase values, 6 resonant frequencies and 6 modal damping values).

Intuitively, information about the current state of the structure must be lost in this data reduction and system identification process. The loss of information occurs primarily from the fact that for a linear system the modal properties are independent of the excitation signal characteristics (amplitude and frequency content) and the location of the excitation whereas the time histories are not. In addition, if the input excites response at frequencies greater than those that can be resolved with the specified data sampling parameters, the identified modes will not provide any information regarding the higher frequency response characteristics of the structure that are contributing to the measured time-history

responses. Within the measured frequency range of response it is often difficult to identify all the modes contributing to the measured response because of coupling between the modes that are closely spaced in frequency. This difficulty is observed more commonly at the higher frequency portions of the spectrum where the modal density is typically greater. Also, the introduction of bias (or systematic) errors, such as those that arise from windowing of the data and those that arise from changing environmental conditions during the test, will tend to make identification of damage based on modal parameters more ambiguous.

#### 4.5.1. Modal Parameter Identification

Standard experimental modal analysis procedures were applied to data obtained from the SET1 accelerometers during the forced vibration tests to identify the modal parameters of the bridge in its damaged and undamaged condition. A rational-fraction polynomial, global, curve-fitting algorithm in a commercial modal analysis software package was used estimate the modal properties. Figure 6 shows the first three modes of the undamaged bridge identified from these data. By measuring the input force and the corresponding driving point acceleration, these mode shapes can be unit-mass normalized.

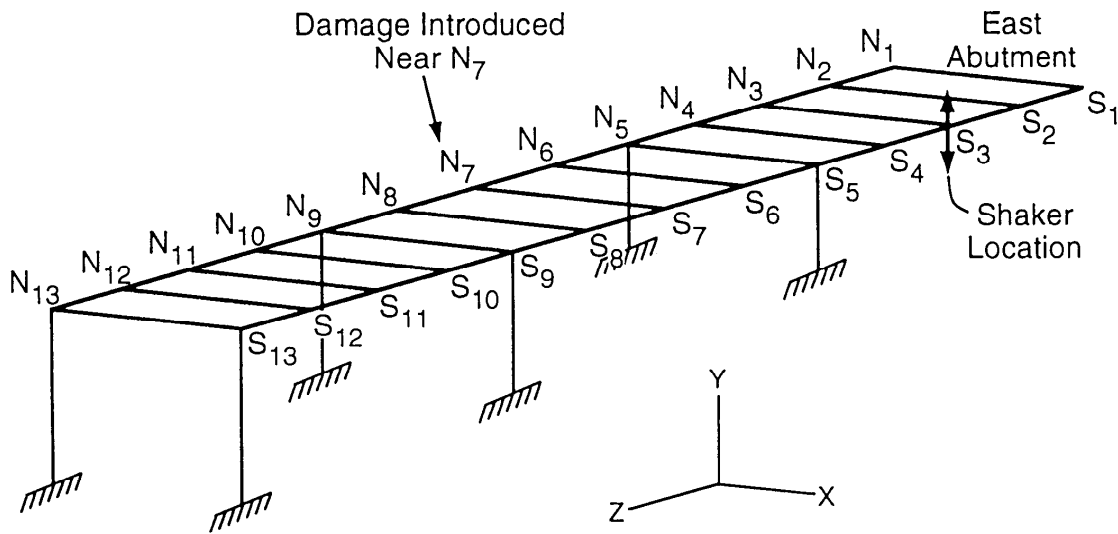


Fig. 4. Set1 (coarse) accelerometer locations.

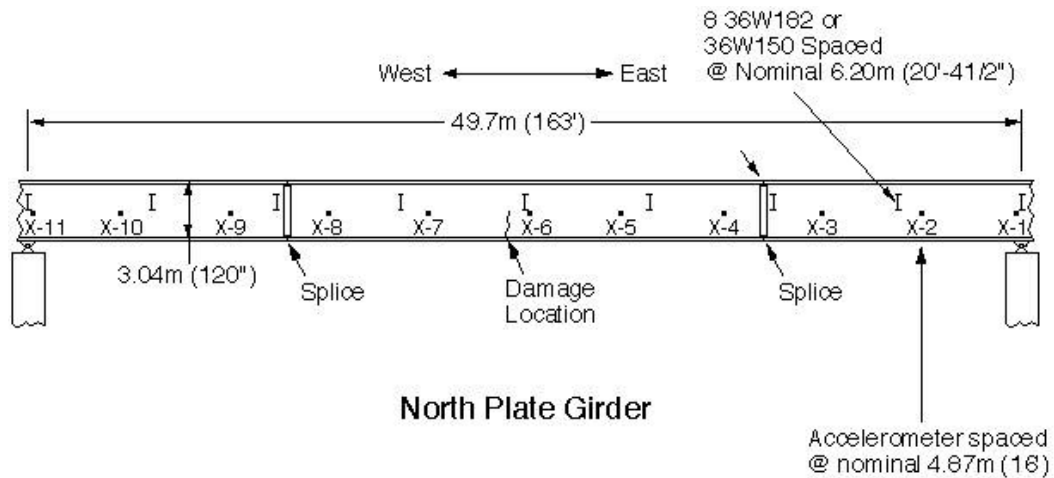


Fig. 5. Set2 (refined) accelerometer locations.

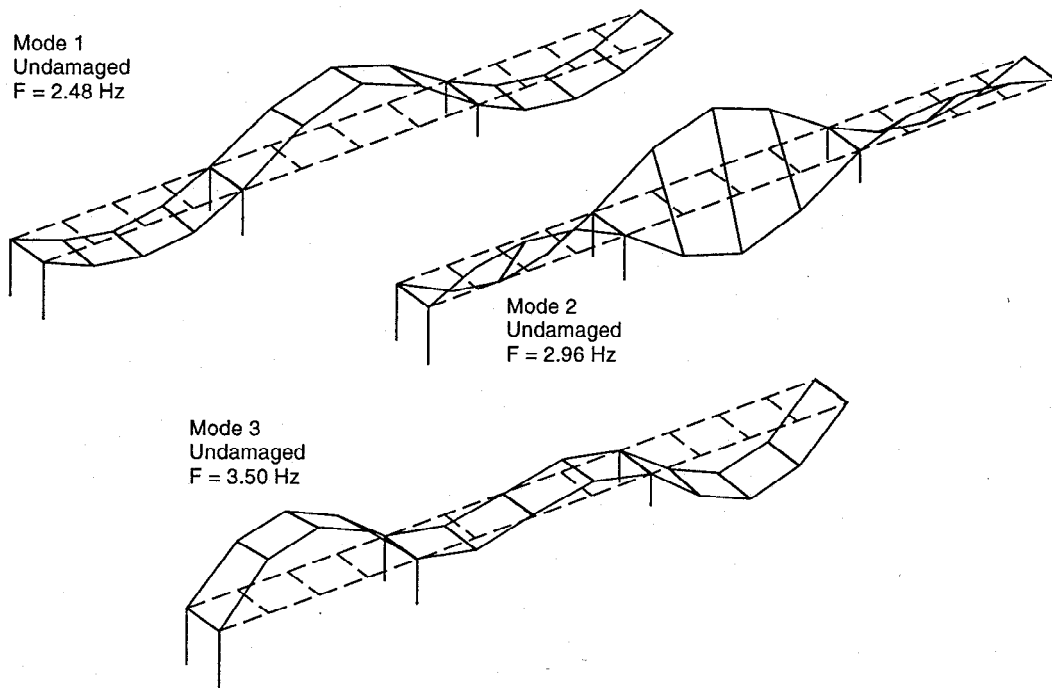


Fig. 6. First three modes measured on the undamaged structure.

Immediately after the forced vibration tests with the SET1 accelerometers were complete, the random excitation tests were repeated using the refined SET2 accelerometers. For these tests the input was not monitored. Operating shapes were determined from amplitude and phase information contained in the cross-power spectra (CPS) of the various accelerometer readings relative to the accelerometer X-3 shown in Fig. 5. Determining operating shapes in this manner, as discussed by (Bendat and Piersol, 1980), simulates the methods that would have to be employed when the responses to ambient traffic excitations are measured. For modes that are well-spaced in frequency these operating shapes will closely approximate the mode shapes of the structure. However, without a measure of the input force these modes cannot be mass normalized.

*Changed in Resonant Frequencies and Modal Damping.* Table 1 summarizes the resonant frequency and modal damping data obtained during each modal test of the undamaged and damaged bridge. No significant changes in the dynamic properties can be observed until the final level of damage was introduced. At the final level of damage the resonant frequencies for the first two modes have dropped to values 7.6 and 4.4 percent less, respectively, than those measured during the undamaged tests. For modes where the damage was introduced near a node for that mode (modes 3 and 5) no significant changes in resonant frequencies can be observed.

*Changes in Mode Shapes.* A modal assurance criterion (MAC), sometimes referred to as a modal correlation coefficient (Ewins, 1985), was calculated to quantify the correlation between mode shapes measured during different tests. Table 2 shows the MAC values that are calculated when mode shapes from tests t17tr (damage level E-1), t18tr (damage level E-2), t19tr (damage level E-3), and t22tr (damage level E-4) are compared to the modes measured on the undamaged forced vibration test, t16tr. The MAC values show no change in the mode shapes for the first three stages of damage. When the final level of damage is introduced, significant drops in the MAC values for modes 1 and 2 are noticed. These two modes are shown in Fig. 7 and can be compared to similar modes identified for the undamaged bridge in Fig 6. When the modes

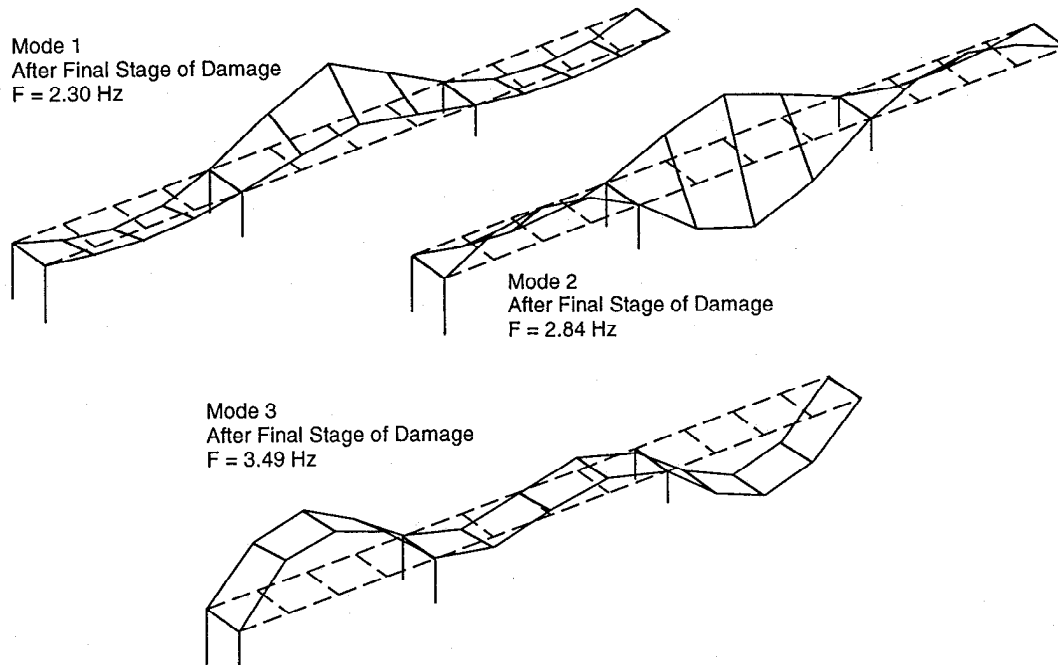


Fig. 7. First three modes measured after the final level of damage.

have a node near the damage location (modes 3 and 5), no significant reductions in the MAC values are observed, even for the final stage of damage. This result corresponds to the observed similarity in mode 3 shown in Figs. 6 and 7.

From the observed changes in modal parameters it is clear that damage can only be definitively identified from change in the modal properties after the final cut was made in the bridge. Prior to the final cut, one could not say that the changes observed were caused by damage or were within the repeatability of the tests. In two tests at increasing levels of damage (t17tr and t18tr) the resonant frequencies were actually found to increase slightly from that of the undamaged case. These slight increases in frequency were measured independently by other researchers studying this bridge at the same time (Farrar, et al. (1994)) and are assumed to be caused by changing test conditions. The examination of changes in the basic modal properties (resonant frequencies and mode shapes) demonstrates the need to identify data features more sensitive to damage.

		Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
Test Designation	Damage Case	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)	Freq. (Hz)/ Damp. (%)
t16tr	Undamaged	2.48/ 1.06	2.96/ 1.29	3.50/ 1.52	4.08/ 1.10	4.17/ 0.86	4.63/ 0.92
t17tr	E-1 cut at center of web	2.52/ 1.20	3.00/ 0.80	3.57/ 0.87	4.12/ 1.00	4.21/ 1.04	4.69/ 0.90
t18tr	E-2 extend cut to bottom flange	2.52/ 1.33	2.99/ 0.82	3.52/ 0.95	4.09/ 0.85	4.19/ 0.65	4.66/ 0.84
t19tr	E-3 bottom flange cut half way	2.46/ 0.82	2.95/ 0.89	3.48/ 0.92	4.04/ 0.81	4.14/ 0.62	4.58/ 1.06
t22tr	E-4 bottom flange cut completely	2.30/ 1.60	2.84/ 0.66	3.49/ 0.80	3.99/ 0.80	4.15/ 0.71	4.52/ 1.06

TABLE 2. Modal Assurance Criteria: Undamaged and Damaged Forced Vibration Tests						
Modal Assurance Criteria		Undamaged (test t16tr) X First level of damage, E-1 (test t17tr)				
Mode	1	2	3	4	5	6
1	0.996	0.006	0.000	0.003	0.001	0.003
2	0.000	0.997	0.000	0.005	0.004	0.003
3	0.000	0.000	0.997	0.003	0.008	0.001
4	0.004	0.003	0.006	0.984	0.026	0.011
5	0.001	0.008	0.003	0.048	0.991	0.001
6	0.001	0.006	0.000	0.005	0.005	0.996
Modal Assurance Criteria		Undamaged (test t16tr) X Second level of damage, E-2, (test t18tr)				
Mode	1	2	3	4	5	6
1	0.995	0.004	0.000	0.004	0.001	0.002
2	0.000	0.996	0.000	0.003	0.002	0.002
3	0.000	0.000	0.999	0.006	0.004	0.000
4	0.003	0.006	0.005	0.992	0.032	0.011
5	0.001	0.006	0.008	0.061	0.997	0.004
6	0.002	0.004	0.000	0.005	0.005	0.997
Modal Assurance Criteria		Undamaged (test t16tr) X Third level of damage, E-3 (test t19tr)				
Mode	1	2	3	4	5	6
1	0.997	0.002	0.000	0.005	0.001	0.001
2	0.000	0.996	0.001	0.003	0.002	0.002
3	0.000	0.000	0.999	0.006	0.006	0.000
4	0.003	0.005	0.004	0.981	0.032	0.011
5	0.001	0.006	0.004	0.064	0.995	0.003
6	0.002	0.002	0.000	0.004	0.009	0.995
Modal Assurance Criteria		Undamaged (test t16tr) X Fourth level of damage, E-4 (test t22tr)				
Mode	1	2	3	4	5	6
1	0.821	0.168	0.002	0.001	0.000	0.001
2	0.083	0.884	0.001	0.004	0.001	0.002
3	0.000	0.000	0.997	0.005	0.007	0.001
4	0.011	0.022	0.006	0.917	0.010	0.048
5	0.001	0.006	0.003	0.046	0.988	0.002
6	0.005	0.005	0.000	0.004	0.009	0.965

#### 4.5.2. Additional Features

For comparative purposes, five linear modal-based damage identification algorithms were applied to data obtained from the I-40 Bridge in its undamaged and damaged condition. These algorithms require mode shape data (in some cases unit-mass normalized mode shape data) and resonant frequencies. A summary of these algorithms and their implementation for the study reported herein can be found in (Farrar and Jauregui, 1996). All five methods are based on the observation that in the vicinity of damage there will be a local increase in the structure's flexibility. This increase will alter the mode shapes of the structure in the damage vicinity, hence a comparison of mode shape data processed by these five different methods, before and after damage, should reveal the location of the damage.

*Damage Index Method.* The Damage Index Method was developed by Stubbs and Kim (1994) to locate damage in structures given their characteristic mode shapes before and after damage. For a structure that can be represented as a beam, a damage index,  $\beta$ , is developed based on the change in strain energy stored

in the structure when it deforms in its particular mode shape. For location  $j$  on the beam this change in the  $i$ th mode strain energy is related to the changes in curvature of the mode at location  $j$ . The damage index for this location and this mode,  $\beta_{ij}$ , is defined as

$$\beta_{ij} = \frac{\left( \int_a^b [\psi_i^{*''}(x)]^2 dx + \int_0^L [\psi_i^{*''}(x)]^2 dx \right) \cdot \int_0^L [\psi_i''(x)]^2 dx}{\left( \int_a^b [\psi_i''(x)]^2 dx + \int_0^L [\psi_i''(x)]^2 dx \right) \cdot \int_0^L [\psi_i^{*''}(x)]^2 dx}, \quad (1)$$

where  $\psi_i''(x)$  and  $\psi_i^{*''}(x)$  are the second derivatives of the  $i$ th mode shape corresponding to the undamaged and damaged structures, respectively.  $L$  is the length of the beam.  $a$  and  $b$  are the limits of a segment of the beam where damage is being evaluated. When more than one mode is used, these authors define the damage index as the sum of damage indices from each mode.

To determine mode shape amplitudes at locations between sensors, the mode shapes are fit with a cubic polynomial. For the refined set of accelerometers, the middle span of the north girder is divided into 160 0.305-m (1-ft) segments. Modal amplitudes are interpolated for each of the 161 nodes forming these segments. Similarly, for the coarse set of accelerometers the entire length of the north girder (all three spans) is divided into 210 0.610 m (2-ft) segments with mode shape interpolation yielding amplitudes at 211 node locations. Statistical methods are then used to examine changes in the damage index and associate these changes with possible damage locations. A normal distribution is fit to the damage indices, and values two or more standard deviations from the mean are assumed to be the most likely location of damage.

*Mode Shape Curvature Method.* Pandey, Biswas, and Samman (1991) assume that structural damage only affects the structure's stiffness matrix and its mass distribution. The pre- and post-damage mode shapes are first extracted from an experimental analysis. Curvature of the mode shapes for the beam in its undamaged and damaged conditions can then be estimated numerically from the displacement mode shapes with a central difference approximation or other means of differentiation. Given the before- and after-damage mode shapes, the authors consider a beam cross section at location  $x$  subjected to a bending moment  $M(x)$ . The curvature at location  $x$  along the length of the beam,  $v''(x)$ , is

$$v''(x) = M(x)/(EI), \quad (2)$$

where  $E$  = the modulus of elasticity, and  $I$  = the moment of inertia of the section.

From Eq. 2, it is evident that the curvature is inversely proportional to the flexural stiffness,  $EI$ . Thus, a reduction of stiffness associated with damage will, in turn, lead to an increase in curvature. Differences in the pre- and post-damage curvature mode shapes will, in theory, be largest in the damaged region. For multiple modes, the absolute values of change in curvature associated with each mode can be summed to yield a damage parameter for a particular location.

*Change in Flexibility Method.* Pandey and Biswas (1994) show that for the undamaged and damaged structure, the flexibility matrix,  $[F]$ , can be approximated from the unit-mass-normalized modal data as follows

$$[F] \approx \sum_{i=1}^n \frac{1}{\omega_i^2} \{\phi_i\} \{\phi_i\}^T \quad \text{and} \quad (3)$$

$$[F]^* \approx \sum_{i=1}^n \frac{1}{\omega_i^{*2}} \{\phi_i\}^* \{\phi_i\}^{*T}, \quad (4)$$

where  $\omega_i$  = the  $i$ th modal frequency,  $\phi_i$  = the  $i$ th unit-mass-normalized mode,  $n$  = the number of measured modes, and the asterisks signify properties of the damaged structure. Equations 3 and 4 are approximations because fewer modes are typically identified than the total numbers of measurement

points or degrees of freedom. From the pre- and post-damage flexibility matrices, a measure of the flexibility change caused by the damage can be obtained from the difference of the respective matrices. The column of the flexibility matrix corresponding to the largest change is indicative of the degree of freedom where the damage is located.

*Change in Uniform Load Surface Curvature.* The coefficients of the  $i$ th column of the flexibility matrix represent the deflected shape assumed by the structure with a unit load applied at the  $i$ th degree of freedom. The sum of all columns of the flexibility matrix represent the deformed shape assumed by the structure if a unit load is applied at each degree of freedom, and this shape is referred to as the uniform load surface. Zhang and Aktan (1995) state that the change in curvature of the uniform load surface can be used to determine the location of damage. In terms of the curvature of the uniform load surface,  $F''$ , the curvature change at location  $l$  is evaluated as follows

$$\Delta F'' = |F_1^{*''} - F_1''|, \quad (5)$$

where  $\Delta F''$  represents the absolute curvature change. The curvature of the uniform load surface can be obtained with a central difference operator as suggested by these authors.

*Change in Stiffness Method.* Zimmerman and Kaouk (1994) have developed a damage detection method based on changes in the stiffness matrix that is derived from measured modal data. The eigenvalue problem of an undamaged, undamped structure is

$$(\lambda_i [M] + [K])\{\psi_i\} = \{0\}, \quad (6)$$

where  $[M]$  is the system mass matrix,  $[K]$  is the system stiffness matrix, and  $\lambda_i$  is the squared natural frequencies corresponding to the modal vector  $\{\psi_i\}$ .

The eigenvalue problem of the damaged structure is formulated by first replacing the pre-damaged eigenvectors and eigenvalues with a set of post-damaged modal parameters (indicated by an asterisk) and, second, subtracting the perturbations in the mass and stiffness matrices caused by damage from the original matrices. Letting  $\Delta M_d$  and  $\Delta K_d$  represent the perturbations to the original mass and stiffness matrices, two forms of a damage vector,  $\{D_i\}$ , for the  $i$ th mode are then obtained by separating the terms containing the original matrices from those containing the perturbation matrices. Hence,

$$\{D_i\} = (\lambda_i^* [M] + [K])\{\psi_i\}^* = (\lambda_i^* [\Delta M_d] + [\Delta K_d])\{\psi_i\}^*. \quad (7)$$

To simplify the investigation, damage is considered to alter only the stiffness of the structure (i.e.  $[\Delta M_d] = [0]$ ). Therefore, the damage vector reduces to

$$\{D_i\} = [\Delta K_d]\{\psi_i\}^*. \quad (8)$$

In a similar manner as the modal-based flexibility matrices previously defined, the approximations to the stiffness matrices, before and after damage, can be used to determine  $[\Delta K_d]$  as

$$[\Delta K_d] = [K] - [K]^* \approx \sum_{i=1}^n \omega_i^2 \phi_i \phi_i^T - \sum_{i=1}^n \omega_i^{*2} \phi_i^* \phi_i^{*T}. \quad (9)$$

A scaling procedure discussed by these authors was used to avoid spurious readings at stiff locations near supports where the signal- to-noise ratio of the measured responses is lower.

*Modifications Made to the Methods to Facilitate Direct Comparison.* The primary modification made to these methods was the adaptation of the cubic polynomial interpolation scheme to approximate mode shape amplitudes at locations between sensors as suggested by Stubbs and Kim (1994). This interpolation effectively introduces artificial degrees of freedom into the experimental measurements. Also, the cubic polynomial can be directly differentiated to obtain the needed mode shape curvatures thus avoiding the finite difference scheme for evaluating the mode shape derivatives suggested by some authors.

Two of the damage detection methods require only consistently normalized mode shapes, namely the Damage Index Method and the Mode Shape Curvature Method. The Change in Flexibility Method, the Change in Uniform Load Surface Curvature Method, and the Change in Stiffness Method require the resonant frequency for each mode and mass-normalized mode shape vectors. When the methods that require unit-mass normalized mode shapes were applied to the mode shape data obtained with the SET2 instruments, the mass along the length of the beam was considered constant and these mode shapes were normalized using an identity matrix. Because these damage detection methods are only concerned with changes in flexibility or stiffness matrices rather than their absolute values, it was assumed that this method of mode shape normalization would not introduce significant errors into the damage detection process.

Tables 3 and 4 summarize the results from applying the five damage detection algorithms to the experimental modal data from the SET1 and SET2 instruments, respectively. In this study, the Damage Index Method was found to have performed the best. All methods were able to definitively locate the damage for the final case, E-4. For the intermediate damage cases mixed results were obtained.

*Model Updating Study.* Simmermacher (1996) applied a finite element model updating procedure to the I-40 Bridge data. The damage detection process essentially parallels that of the change in stiffness method previously discussed. However, the stiffness matrix was obtained from a finite element model that had been correlated with the measured modal parameters. Three correlation procedures were used. The minimum rank perturbation method (MRPT) was applied to update only the stiffness matrix and then to update both the mass and stiffness matrix. Next, a method for updating the stiffness matrix proposed by Baruch and Bar Itzhack (1978) was also investigated. Two different finite element models of different complexity were studied. Both models required considerable degree of freedom (DOF) reduction so that the model DOF corresponded to the measurement DOF. Four different reduction procedures were studied and that proposed by Guyan (1965) was found to produce the best results.

For a model with 78 DOF all methods were able to locate damage for Case E-4. The Baruch method was able to locate damage for case E-3. Neither method could locate the damage for case E-2. The authors point out that for this model the Baruch method has the advantage that it produces minimal changes to the stiffness matrix such that the model agrees with the test data. The MRPT methods do not preserve the proper physical stiffness distribution in the updated model. For the model with 1422 DOF, which required 99% of the DOF to be removed in the reduction process, the MRPT method could locate the damage for Case E-4. The Baruch method did not locate any damage cases properly. In this case the original model was so poor that large changes to the stiffness matrix were required. The Baruch method did not adjust the model sufficiently to account for these necessary changes.

TABLE 3. Summary of Damage Detection Results using Experimental Modal Data (SET1)				
Damage Cases				
Damage ID Method	E-1	E-2	E-3	E-4
Damage Index Method	**	**	**	*
Mode Shape Curvature Method	**	*	**	*
Change in Flexibility Method	○	○	**	*
Change in Uniform Load Surface Curvature Method	○	○	○	*
Change in Stiffness Method	**	**	**	*
* Damage located, ** Damage located using only 2 modes; ○ Damage not located				

TABLE 4. Summary of Damage Detection Results using Experimental Modal Data (SET2)				
Damage Cases				
Damage ID Method	E-1	E-2	E-3	E-4
Damage Index Method	●	●	●	●
Mode Shape Curvature Method	●●●	●●	●	●
Change in Flexibility Method	○	○	○	●
Change in Uniform Load Surface Curvature Method	○	●●●	●	●
Change in Stiffness Method	○	○	○	●
● Damage located, ●● Damage narrowed down to two locations, ●●● Damage narrowed down to three locations, ○ Damage not located				

Finally, a study of the effects of noise on the damage detection process was undertaken by adding noise to the modal vectors. The sensitivity of the various measurement locations to damage was studied. It was found that the locations at the center of the middle span were the most sensitive to noise and one of these locations correspond to the point where damage was introduced. Subsequent applications of the MRPT method using the noisy modes revealed that the addition of noise actually enhanced the damage detection process because the damage was located at a point very susceptible to the influence of noise.

#### 4.6. STATISTICAL MODEL BUILDING

Exclusive of the procedure used in the damage index method to identify the locations that show the largest change in the damage index statistical models were not employed in this study. The distinct need for statistical analysis of the data and quantification of the environmental effects on the measured modal properties is evident when one considers the small changes in measured dynamic response that are being examined. The topic of variability in modal properties and statistical analysis of the I-40 Bridge data are discussed in more detail in (Doebling and Farrar, 1998) and (Doebling and Farrar, 1997). Performing statistical analysis of the dynamic data is imperative if one is to establish that changes brought about by damage are greater than the test-to-test repeatability. Methods for performing such statistical analyses are summarized in (Doebling, Farrar, and Goodman, 1997) and (Farrar, Doebling and Cornwell, 1998).

Although the number of papers reporting vibration-based damage detection results from a variety of structures has greatly increased in recent years, very few of the articles examine the variability in the modal properties that can arise from changes in environmental conditions or from random and systematic errors inherent in the data acquisition/data reduction process. A thorough study of the modal parameter variability must be conducted before vibration-based damage id algorithms can be applied with confidence.

#### 4.7. IN HIND SIGHT, THINGS THAT SHOULD HAVE BEEN DONE DIFFERENT

Based on subsequent analysis and observations related to the I-40 bridge tests (Doebling and Farrar, 1997 and 1998), subsequent tests on another bridge (Doebling Farrar and Goodman, 1997) and (Farrar Doebling and Cornwell, 1998), interactions with other researchers in the field (particularly those at the Univ. of Cincinnati and Drexel Univ.), and review of the technical literature related to bridge testing, there are several things that should have been done during these tests (and have been done on subsequent tests) to improve the confidence in the damage ID results. These improvements are listed below.

##### 4.7.1. *Perform a More Thorough Pre-test Visual Inspection*

In addition to vibration tests visual inspection is needed to ascertain the initial condition of the structure. Particular attention should be paid to boundary conditions and changes to the neighboring vicinity of the test structure. During the tests visual inspection revealed that the east end of the top portion of the south

girder were not in contact with the concrete at the top of the abutment, but very close to it. During earlier tests observations of the end condition were not made and it was speculated that during cooler weather the girders could have actually been in contact with the abutment.

#### *4.7.2. Perform Linearity and Reciprocity Checks*

The system identification portion of the experimental modal analysis procedure typically relies on the assumption that the structure is linear. Linearity can be checked, to some degree, by exciting the structure at different levels and overlaying the measured FRFs for a particular point. Ideally, with the thought of an on-line monitoring system in mind, these different excitation levels would span the range of loading observed during ambient traffic vibration measurements. Also, a change in the linearity properties can in itself be an indication of damage. In addition to the assumption of linearity, the system identification portion of the experimental modal analysis typically relies on the assumption that the structure will exhibit reciprocity. Performing a reciprocity check is much more involved when a large shaker is being used because of the setup time involved in relocating the shaker. Also, to check the reciprocity of the structure alone, one must relocate the accelerometers and cables as well as the shaker (Farrar, Doebling, Cornwell and Straser, 1997). Without moving the instrumentation, the reciprocity check will involve reciprocity of the electronics as well as that of the structure. Because, in general, the electronics will not be moved once the test has started, the latter test is more representative of the reciprocity of the system.

#### *4.7.3. Perform as Many Environmental and Testing Procedure Sensitivity Studies as Possible*

Sensitivity of modal test results to environmental conditions and test procedures should be quantified to the extent possible. Subsequent tests (after the potential damage has occurred) should be performed under similar environmental conditions using similar test procedures, if possible. Figure 8 shows the change in modal frequencies measured on another bridge as a function of the temperature differential across the deck (Farrar, Doebling, Cornwell and Straser, 1997). Changes in the first mode frequency of five-percent are noted over a 24 Hr time span. These changes are comparable to the changes in resonant frequency resulting from the final level of damage in the I-40 Bridge tests. Also, a baseline noise measurement should be made for the data acquisition system.

#### *4.7.4. Perform False-positive Studies*

As a means of quantifying the effects observed in the sensitivity studies, sets of data from the undamaged structure should be analyzed with the damage id algorithm to demonstrate that the algorithm will not falsely predict damage when in fact none has occurred.

### **4.8. WHAT CAN BE DONE WITH ONLY INITIAL MEASUREMENTS AND FEM?**

Once a finite element analysis has been benchmarked or correlated against the measure modal properties simulated damage scenarios can be introduced into the model and either an eigenvalue analysis can be performed or, to better simulate an actual modal test, a time-history analysis can be performed. Mode shape data can then be obtained from either type of analysis and the various damage ID methods can be applied to the observed changes in the modal properties. If a statistical analysis has been applied to the measured modal parameters of the baseline or undamaged structure, then it can be established that the changes in the monitored modal properties such as mode shape curvature resulting from the simulated damage are greater than the variations that can be attributed to experimental repeatability. In addition, the statistical variations calculated for the measured modal properties on the undamaged structure can be assumed to apply to the numerical results from the damaged structure. The use of statistical variations measured on the undamaged structure and assumed for the numerical simulation of the damaged structure can then be used to establish the threshold damage level that can be reliably detected.

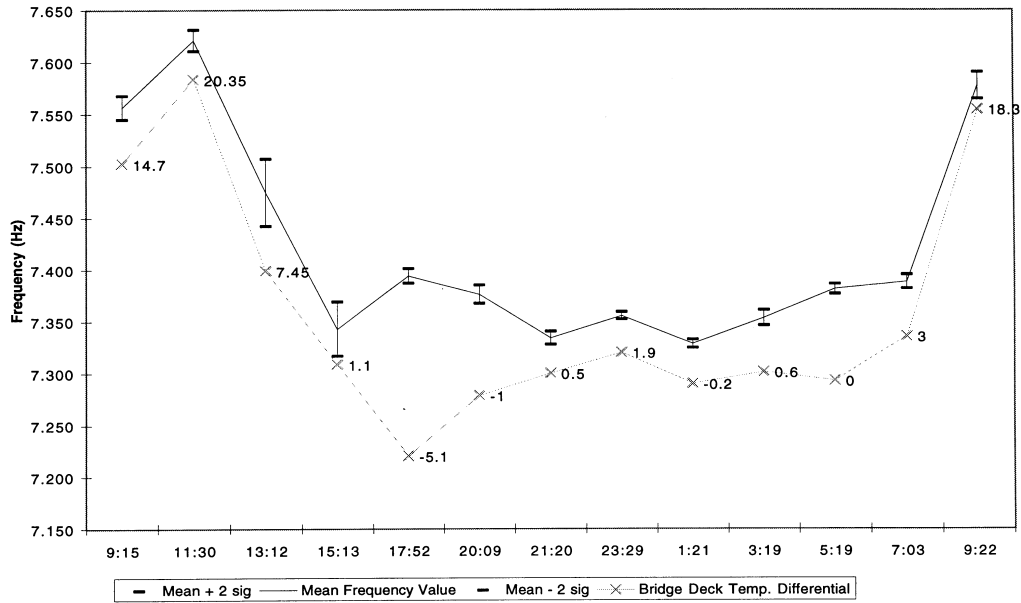


Fig. 8. Change in the first mode frequency during a 24 hr time period

## 5. Application of Vibration-based Damage Detection to Concrete Bridge Columns

A damage detection study was conducted on two concrete columns that were quasi-statically loaded to failure in an incremental manner. The focus of this study was to establish a relatively simple feature vector coupled with a simple statistical model that would unambiguously identify that the columns had been damaged.

### 5.1. TEST STRUCTURE GEOMETRY

The test structures consisted of two 61-cm-dia (24-in-dia) concrete bridge columns that were subsequently retrofitted to 91-cm-dia (36-in-dia) columns. Figure 2 shows the test structure geometry. The first column tested, labeled Column 3, was retrofitted by placing forms around the existing column and placing additional concrete within the form. The second column, labeled Column 2, was extended to the 91-cm-diameter by spraying concrete in a process referred to as shotcreting. Column 2 was then finished with a trowel to obtain the circular cross-section.

The 91-cm-dia. portion of both columns were 345 cm (136 in) in length. The columns were cast on top of a 142-cm-sq. (56-in-sq.) concrete foundation that was 63.5-cm (25-in) high. A 61-cm-sq. concrete block that had been cast integrally with the column extends 46-cm (18-in.) above the top of the 91-cm-dia. portion of the column. This block was used to attach the hydraulic actuator to the columns for quasi-static cyclic testing and to attach the electro-magnetic shaker used for the experimental modal analyses. As is typical of actual retrofits in the field, a 3.8-cm-gap (1.5-in-gap) was left between the top of the foundation and the bottom of retrofit jacket. Therefore, the longitudinal reinforcement in the retrofitted portion of the column did not extend into the foundation. The concrete foundation was bolted to the 0.6-m-thick (2-ft-thick) testing floor in the University of California-Irvine structural-testing laboratory during both the static cyclic tests and the experimental modal analyses. The structures were not moved once testing was initiated.

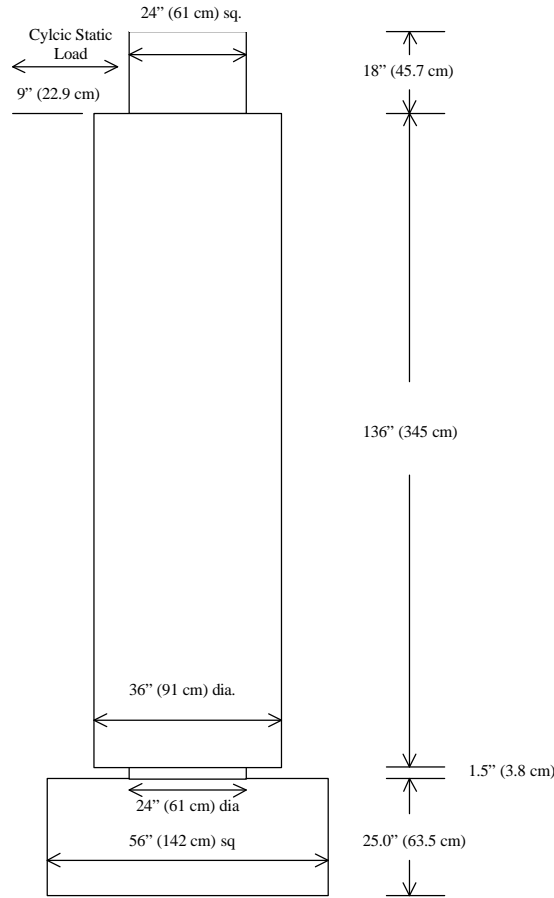
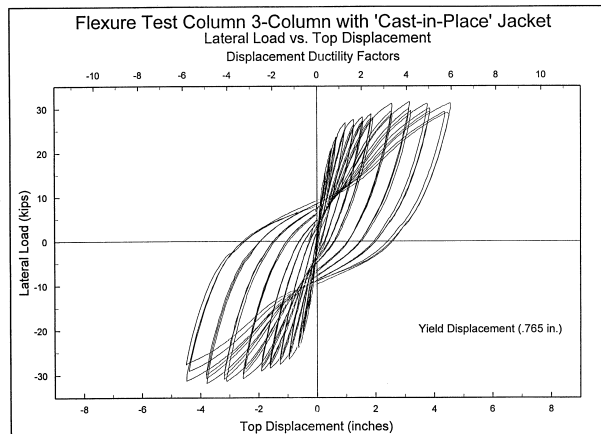


Fig. 9 Column Dimensions and photo of an actual test structure.

## 5.2. QUASI-STATIC LOADING

Prior to applying lateral loads, an axial load of 400 kN (90 kips )was applied to simulate gravitational loads that an actual column would experience. Next, a hydraulic actuator was used to apply lateral load to the top of the column in a cyclic manner. The loads were first applied in a force-controlled manner to produce lateral deformations at the top of the column corresponding to  $0.25\Delta y_T$ ,  $0.5\Delta y_T$ ,  $0.75\Delta y_T$  and  $\Delta y_T$ . Here  $\Delta y_T$  is the lateral deformation at the top of the column corresponding to the theoretical first yield of the longitudinal reinforcement. The structure was cycled three times at each of these load levels.

Based on the observed response, a lateral deformation corresponding to the actual first yield,  $\Delta y$ , was calculated and the structure was cycled three times in a displacement-controlled manner to that deformation level. Next, the loading was applied in a displacement-controlled manner, again in sets of three cycles, at displacements corresponding to  $1.5\Delta y$ ,  $2.0\Delta y$ ,  $2.5\Delta y$ , etc. until the ultimate capacity of the column was reached. Load deformation curves for Column 3 are shown in Fig 10. This manner of loading put incremental and quantifiable damage into the structures. The axial load was applied during all static tests.



### 5.3. DYNAMIC EXCITATION

For the experimental modal analyses the excitation was provided by an electro-magnetic shaker mounted off-axis at the top of the structure. The shaker rested on a steel plate attached to the concrete column. Horizontal load was transferred from the shaker to the structure through a friction connection between the supports of the shaker and the steel plate. This force was measured with an accelerometer mounted to the sliding mass ( $0.18 \text{ lb-s}^2/\text{in}$  (31 Kg)) of the shaker. A 0 - 400 Hz uniform random signal was sent from a source module in the data acquisition system to the shaker but feedback from the column and the dynamics of the mounting plate produced an input signal that was not uniform over the specified frequency range.

### 5.4. OPERATIONAL EVALUATION

Because the structure being tested was a laboratory specimen, operational evaluation was not conducted in a manner that would typically be applied to an *in situ* structure. However, the vibration tests were not the primary purpose of this investigation. Therefore, compromises had to be made regarding the manner in which the vibration tests were conducted. The primary compromise was associated with the mounting of the shaker. These compromises are analogous to operational constraints that may occur with *in situ* structures. Environmental variability was not considered an issue because these tests were conducted in a laboratory setting. The available measurement hardware and software placed the only constraints on the data acquisition process.

### 5.5. DATA ACQUISITION AND CLEANSING

Forty accelerometers were mounted on the structure as shown in Fig. 11. These locations were selected based on the initial desire to measure the global bending, axial and torsional modes of the column. Note that the accelerometers at locations 2, 39 and 40 had a nominal sensitivity of 10mV/g and were not sensitive enough for the measurements being made. As part of the data cleansing process, data from these channels were not used in subsequent portions of the damage detection process. Locations 33, 34, 35, 36, and 37 were accelerometers with a nominal sensitivity of 100mV/g. All other channels had accelerometers with a nominal sensitivity of 1V/g.

A commercial data acquisition system was used to record and digitize all accelerometer signals. Data acquisition parameters were specified to obtain 8-s-duration time-histories discretized with 8192 points. Only one average was measured. A uniform window was specified for these data, as the intent was to measure a time history only.

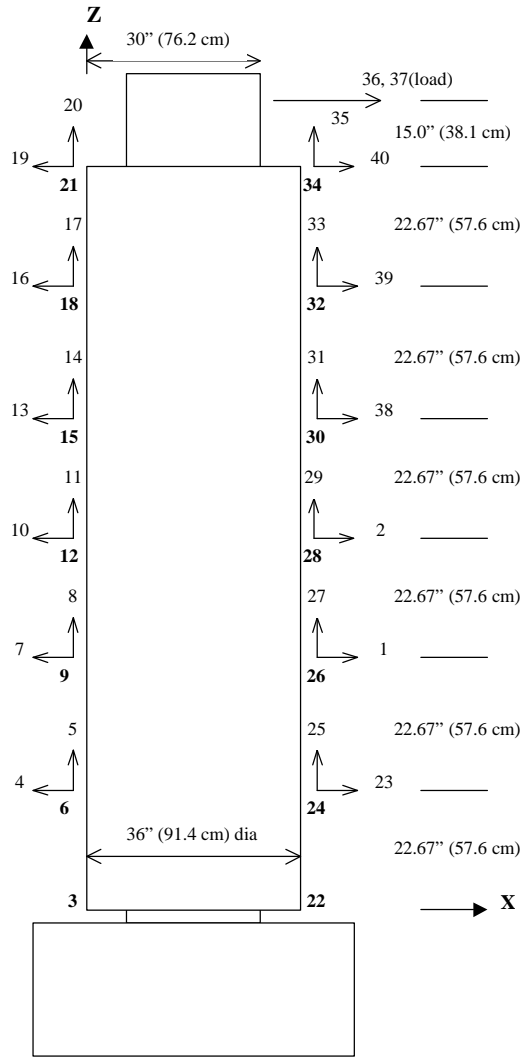


Fig. 11 Accelerometer locations and coordinate system for modal testing. Accelerometers 3, 6, 9, 12, 15, 18, 21, 22, 24, 26, 28, 30, 32, and 34 are mounted in the  $-y$  direction.

## 5. 6. FEATURE SELECTION

Typically, systematic differences between time series from the undamaged and damaged structures are nearly impossible to detect by eye. Therefore, other features of the measured data must be examined for damage detection. Originally, damage detection features were to be based on common modal properties as have been done in many previous studies. However, the feedback from the structure and mounting system to the shaker produced an input that did not have a uniform power spectrum over the frequency range of interest as previously discussed. This input form coupled with the nonlinear response observed at higher levels of damage made it extremely difficult to track changing modal properties through the various levels of damage. Therefore, other features were sought for the damage detection process.

The alternate features were selected based on previous experience from speech pattern recognition where auto- regressive models have been used to estimate the transfer function of the human vocal track

(Morgan and Scofield, 1992). The time series were modeled using a common method of auto-regressive estimation also referred to as Linear Predictive Coding (LPC) (Rabiner and Shafe, 1978). The LPC algorithm is an  $N$ th-order model that attempts to model the current point in a time series,  $x'(t_n)$ , as a linear combination of the previous  $N$  points. That is

$$x'(t_n) = \sum_{i=1}^N a_i x(t_{n-i}) \quad (10)$$

Third-order LPC models were developed for each column using 512-point windows with 97% overlap resulting in 480 samples of the  $a_i$ 's. Over these segments of the time series the  $a_i$ 's that best model the time series in a least squares sense are used as features that are assumed to be representative of the system's dynamic response during those samples. Hanning windows were applied to these data prior to the estimate of the coefficients. These models were developed with data from sensors 3 and 21 (Fig. 11). Sensor 3 was located close to the damage, but because of the test configuration this sensor was not expected to experience large amplitude response as it primarily measures torsional motion of the structure near its fixed end. Sensor 21 was located farther from the damage and experienced some of the largest amplitude response as it primarily measured the bending response at the free end of this cantilever structure.

Over a time series, many overlapping "windows" give rise to LPC coefficient vectors, which become the multi-dimensional data samples to be analyzed in the statistical model development portion of the damage detection process. While the overlapping of windows provides a smoother estimate of the features' changes over time, samples that result from overlapping windows will not be independent.

Normalization of the data was not attempted because these tests were conducted in a laboratory environment where the input could be applied in a very controlled manner. Other considerations that led to the decision not to normalize the data included the consideration that environmental and test-to-test variability was negligible, damage was introduced in discrete increments, and it was assumed that the vibration levels were such that the physical condition of the test structures did not change during the dynamic tests.

## 5. 7. STATISTICAL MODEL DEVELOPMENT: FISHER'S DISCRIMINANT

Consider two data generation processes A and B, with independent multi-dimensional samples  $\{x\}$  being generated by both processes. Assuming A and B have some systematic difference in the samples that they generate, Fisher's discriminant (Fisher, 1936, and Bishop, 1995) represents the optimal linear projection of the multidimensional sample space that maximally discriminates the  $\{x_A\}$ 's from the  $\{x_B\}$ 's. That is, it defines a linear projection  $\{w\}$  such that

$$y = \{w\}^T \{x\} \quad (11)$$

produces a scalar projection,  $y$ , of the multidimensional space onto which the distribution of  $\{x_A\}$ 's is as distinct as possible from the distribution of  $\{x_B\}$ 's. Once this projection is determined from previous samples of  $\{x_A\}$ 's and  $\{x_B\}$ 's it can be used to provide the relative probability that a novel sample  $\{x\}$  was generated by process A or B. Thus, the Fisher discriminant maximizes the function  $F(\{w\})$ , which is the distance between the means of the transformed distributions,  $\mu_i$ , normalized by the total within-class covariance,  $s_k^2$ :

$$F(\{w\}) = \frac{(\mu_A - \mu_B)^2}{s_A^2 + s_B^2}, \quad (12)$$

Equation 12 can be rewritten explicitly in terms of  $\{w\}$  as

$$F(\{w\}) = \frac{\{w\}^T [S_b] \{w\}}{\{w\}^T [S_w] \{w\}}, \quad (13)$$

where  $[S_b]$  is the between-class covariance matrix, and  $[S_w]$  is the total within-class covariance matrix.

Once the data have been projected down onto the scalar  $y$  dimension, the distribution of  $y_A$  and  $y_B$  points can be described by an appropriate probability density function. Since it was originally assumed that  $\{x\}$  was a multi-dimensional random variable, then  $y = \{w\}^T \{x\}$  is a sum of random variables and the central limit theorem is invoked to justify modeling  $y_A$  and  $y_B$  with Gaussian density functions. Novel data  $\{x_{new}\}$  can be projected to get  $y_{new} = \{w\}^T \{x_{new}\}$  and the likelihood,  $p$ , of  $y_{new}$  with respect to the Gaussian for class A and the Gaussian for class B can be determined.

### 5.8. APPLICATION OF FISHER'S DISCRIMINANT TO CONCRETE COLUMN DATA

Fisher's discriminant was defined using data from the vibration tests conducted on the undamaged columns and from the vibration tests conducted after the first level of damage corresponding to initial yielding of the steel reinforcement. Subsequent damage levels were then identified based on this same Fisher projection. As illustrated in Fig. 13, when Fisher's discriminant is applied to data from both sensors on either column, there is statistically significant separation between the LPC coefficients for the undamaged cases and damage level 1 cases (solid and dashed Gaussian density functions). Also plotted as straight lines are the results of using the previously determined Fisher projection to project many samples of data from increasingly greater levels of damage into this space. While increasing damage is not necessarily related to increasing Fisher coordinate, all damaged cases have a profile significantly different from that of the undamaged case. The discrimination between damaged and undamaged structures is obtained with both with data from both sensors. This result is significant because the response measured by Sensor 3 was of relatively low amplitude with noise contributing significantly to the measured signal. Higher-order LPC models and different size data windows produced similar results.

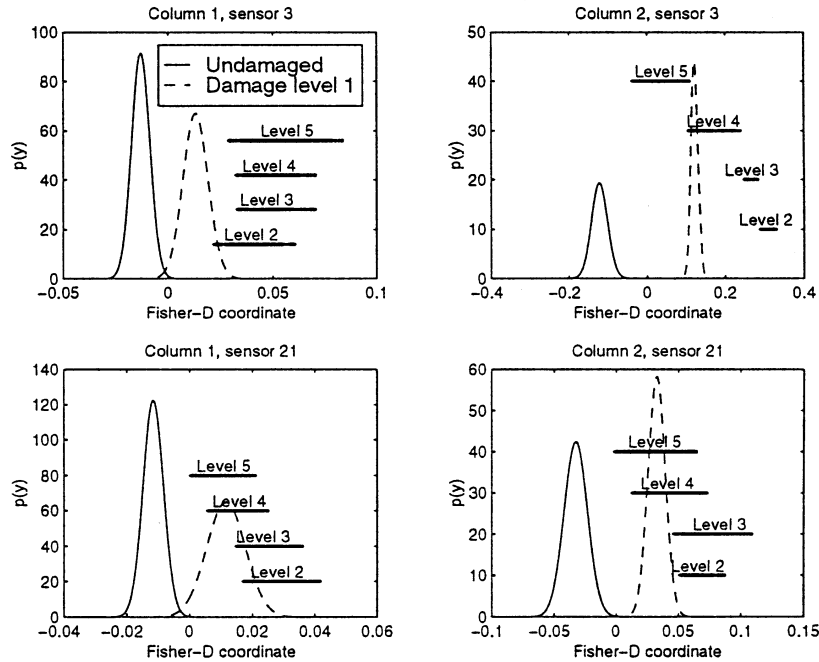


Fig. 13 Distribution of LPC-generated feature vectors projected onto the Fisher coordinate. Solid horizontal lines represent widths of the distributions for higher damage levels.

## 6. Concluding Comments

Recent work in structural health monitoring and vibration-based damage detection has been briefly reviewed to show that this subject is the focus of many active research efforts and to identify some of the technical challenges in this field. A major shortcoming associated with many of these efforts is that statistical models are not applied to identify when changes in the selected features are significant. Therefore, a statistical-pattern-recognition paradigm has been proposed for the general problem of structural health monitoring. This paradigm breaks the process of structural health monitoring into the four tasks of operational evaluation, data acquisition and cleansing, feature selection, and statistical model development. Structural damage detection studies of the I-40 Bridge and of concrete columns subjected to quasi-static cyclic loading to failure were then posed in terms of this paradigm.

The application of linear damage identification methods using experimental modal data gathered from the I-40 Bridge over the Rio Grande in Albuquerque, NM was first summarized. In this study linear damage identification implies that linear dynamic models were used to model the structure both before and after damage. The nature of the damage applied to the I-40 Bridge was such that the linear damage models are applicable to these damage scenarios.

Examination of results from the experimental modal analyses verify other investigators findings that standard modal properties such as resonant frequencies and mode shapes are poor indicators of damage. The more sophisticated damage detection methods investigated herein showed improved abilities to detect and locate the damage. In general, all methods investigated in this study identified the damage location correctly for the most severe damage case; a cut through more than half the web and completely through the bottom flange. However, for several of these methods, if they had been applied blindly, it would be difficult to tell if damage had not also occurred at locations other than the actual one. The methods were inconsistent and did not clearly identify the damage location when they were applied to the less severe damage cases. The authors feel that the summary of this application to a large, *in situ* structure demonstrates the positive aspects as well as drawbacks of this emerging technology.

The I-40 Bridge tests highlight the fact that damage typically is a local phenomenon. Local response is captured by higher frequency modes whereas lower frequency modes tend to capture the global response of the structure and are less sensitive to local changes in a structure. From a testing standpoint it is more difficult to excite the higher frequency response of a structure as more energy is required to produce measurable response at these higher frequencies than at the lower frequencies. These factors coupled with the loss of information resulting from the necessary reduction of time-history measurements to modal properties add difficulties to the process of damage identification based on standard modal properties. These factors contribute to the current state where this technology is still in the research arena with only limited applications to large civil engineering infrastructure.

Based on further analysis and observations related to the I-40 bridge tests, subsequent tests on another bridge, interactions with other researchers performing similar tests, and review of the technical literature related to bridge testing, there are several things that should have been done during these tests to improve the confidence in the damage ID results. These improvements include detailed visual inspection of the bridge, performing linearity checks, performing reciprocity checks, performing false-positive studies, performing test condition sensitivity studies, and performing statistical analyses of the measured modal properties.

The results of a damage detection study applied to reinforce concrete bridge piers were then summarized. This study attempted to identify the relatively simple features of the measured data that were sensitive to damage. Criteria for selecting the features were to keep the dimension of the feature vector small and have the number of samples of the vector large. The feature vectors used were the coefficients of a third-order linear predictive coding model. A well-developed procedure for group classification, the linear discriminant operator referred to as "Fisher's Discriminant", was introduced for application this vibration-based damage detection problem. This procedure requires data to be available from both the undamaged and damaged structures.

The results of this study indicate a strong potential for using linear discriminant operators to identify the presence of damage. An attractive attribute of this statistical model is that it was applied to features

obtained from response data only implying that is appropriate for structures subjected to ambient vibration from sources such as traffic or wind excitation.

The results of this study also suggest that if one or more common forms of damage occur, it may be possible to not only determine that a system is damaged but to determine which form of damage has occurred. Additional data is required to explore this possibility. Another attractive feature of the linear discriminant operator that was not fully explored during this investigation is its ability to combine data from various types of sensors. This feature will become particularly attractive when monitoring structures that experience significant variations in their dynamic response resulting from changing environmental and operating conditions. Further analyses are also required to demonstrate the ability of the linear discriminant operator to avoid false-positive indications of damage. However, multiple samples of data from the undamaged columns were not measured.

To advance the state of the art in vibration-based damage detection it is the authors' opinion that developments of non-model based pattern recognition methods will be needed to supplement the existing model-based techniques. It is anticipated that such methods will be particularly effective when analyzing a structure where the damage changes the structure from a predominantly linear system to a predominantly nonlinear system.

Data from the I-40 Bridge study and the study of the concrete columns can be downloaded from Los Alamos National Laboratory's Damage ID Web Page: [http://ext.lanl.gov/projects/damage\\_id](http://ext.lanl.gov/projects/damage_id).

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